# PRELIMINARY PROPOSED SEISMIC DESIGN AND EVALUATION CRITERIA FOR NEW AND EXISTING UNDERGROUND HAZARDOUS MATERIALS STORAGE TANKS

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## 1 Basic Criteria

This document provides a recommended set of deterministic seismic design and evaluation criteria for either new or existing underground hazardous materials storage tanks placed in either the high hazard or moderate hazard usage categories of UCRL-15910 (Ref. 1). The criteria given herein are consistent with and follow the same philosophy as those given in UCRL-15910 for the U.S. Department of Energy facilities. This document is intended to supplement and amplify upon Reference 1 for underground hazardous materials storage tanks.

Reference 1 aims at achieving an annual probability of seismic-induced damage beyond which hazardous material confinement is impaired of about $10^{-5}$ and $10^{-4}$ for new high hazard and moderate hazard usage categories respectively. These seismic performance goals are achieved by defining the seismic hazard in terms of a site-specific design response spectrum (called herein, the design basis earthquake (DBE), which has a "best-estimate" annual frequency of exceedance of about $2x10^{-4}$ for high hazard facilities and $1x10^{-3}$ for moderate hazard facilities coupled with adequately conservative deterministic acceptance criteria. To be adequately conservative, the acceptance criteria must introduce approximately a factor of 20 further reduction for high hazard facilities (10 for moderate hazard facilities) in the risk of unacceptable performance below the annual risk of exceeding the DBE.

If an existing facility is close to meeting the guidelines, up to a doubling in the annual seismic risk is allowed by Reference 1 because (1) such a doubling represents only a minor adjustment in the approximate seismic performance goals and in the resulting acceptance criteria, (2) existing facilities may have a shorter remaining life than a new facility, and (3) it is far more difficult to upgrade an existing facility compared to incorporating increased resistance in a new design. This relief is not automatic, but must be justified on a case-by-case basis. Reference 1 presents two acceptable methods to achieve this relief. One method to achieve this relief is to reduce the conservatism in the acceptance criteria by an amount roughly corresponding to a doubling of the risk of unacceptable seismic performance beyond that for new facilities. The other method to achieve this relief is to maintain the conservatism specified in the acceptance criteria for new facilities and to reduce the DBE by accepting a doubling of its annual frequency of exceedance.

In summary, Reference 1 define the following steps for establishing seismic design or evaluation criteria:

1. Establish an acceptable approximate seismic performance goal for the components being designed or evaluated. For new and existing high hazard and moderate hazard usage categories, Reference 1 suggests seismic performance goal probability, $P_F$, in the range of $1x10^{-5}$ to $2x10^{-4}$ annual probability of unacceptable performance.

2. Establish a set of conservative seismic acceptance criteria which introduce a significant reduction in the risk of unacceptable seismic performance below the annual frequency of exceedance of the DBE. Reference 1 presents general seismic acceptance criteria aimed at achieving seismic risk reduction facors, R, of about 5, 10, and 20.

3. Establish the DBE at an annual frequency of exceedance equal to R (from Step #2) times $P_F$ (from Step #1).

These steps are general in nature, and can be followed with different seismic performance goal probabilities than those suggested in Reference 1.

This document provides seismic design and evaluation criteria for underground hazardous material storage tanks aimed at achieving seismic risk reduction factors, R, of either 5, 10, or 20. A base seismic capacity, $C_{SB}$ criteria is presented for R of 20. This criteria is then modified by appropriate scale factors for R of 5 and 10. The criteria presented are primarily based upon the judgement and experience of the author as being appropriate to roughly achieve these seismic risk reductions. However, great rigor or quantitative accuracy in achieving these seismic risk reduction factors should not be implied. The factors merely served as target goals when developing this criteria.

For the typical probabilistic seismic hazard curve, an order of magnitude reduction in the "best-estimate" annual frequency of exceedance from $1x10^{-4}$ to $1x10^{-5}$ corresponds to an approximate doubling of the ground motion. Figure 1 presents a typical probabilistic hazard curve expressed in terms of peak ground acceleration (PGA) that has this typical characteristic of a doubling of the ground motion from $1x10^{-4}$ to $1x10^{-5}$. Figure 1 is normalized to a PGA of 0.30g at $1x10^{-4}$. However, the PGA amplitude is unimportant, only the shape of this curve is important. The seismic acceptance criteria presented herein were developed to roughly achieve the seismic performance goals of Reference 1 (summarized above) for probabilistic seismic hazard curves with shapes similar to Figure 1, i.e., an approximate doubling of the ground motion for an order of magnitude reduction in the annual probability of exceedance from$1x10^{-4}$ to $1x10^{-5}$.

Within this document, the base seismic capacities $C_{SB}$ are established sufficiently conservative to achieve roughly a factor of 20 further reduction in annual risk of unacceptable seismic performance below the annual probability of the DBE for the Figure 1 probabilistic hazard curve shape. As can be demonstrated, this reduction is achieved when the base seismic capacity $C_{SB}$ is set at a factor of safety of 2.0 below the estimated 10 percent failure probability capacity $C_{10\%}$, i.e.:

$$C_{SB} = \frac{C_{10\%}}{2.0} \tag{1}$$

The deterministic seismic capacity approach defined herein is roughly aimed at meeting Equation (1). For reduction factors other than 20, the deterministic capacity $C_S$ is obtained by scaling upward $C_{SB}$ by the appropriate scale factor $S_F$, or:

$$C_S = S_F \ C_{SB} \tag{2}$$

where:

$S_F$ = 1.0 for reduction of 20
$S_F$ = 1.2 for reduction of 10
$S_F$ = 1.50 for reuction of 5

This basic seismic criteria is summarized in Table 1.

The scale factors shown above are appropriate for seismic hazard curves similar to Figure 1 for which a doubling of the ground motion results in approximately an order of magnitude reduction in the annual frequency of exceedance over the important annual frequency range from $1x10^{-3}$ to $1x10^{-5}$. Defining $a_3$ and $a_5$ as the ground motion values with exceedance frequencies of $10^{-3}$ and $10^{-5}$, respectively, the $a_5/a_3$) may be used as a measure of the hazard curve slope within this annual frequency range of greatest interest. For Figure 1:

$$(a_5/a_3) = \frac{0.60}{0.14} = 4.29$$

It can be shown that within a broad range of $(a_5/a_3)$, the scale factors are not sensitive to the hazard slope factor $(a_5/a_3)$ or the fragility logarithmic standard deviation ß of the component being evaluated. The above scale factors are accurate within 10 percent for $(a_5/a_3)$ ratios between 3.5 and 5.0 and component fragility logarithmic standard deviations between 0.3 and 0.5. All component fragility logarithmic standard deviations are expected to lie within this range and most seismic hazard curve slopes are also within the above range. Ratios of $(a_5/a_3)$ greater than 5.0 are not considered credible. However, $(a_5/a_3)$ ratios down to about 2.2 can occur at western United States sites with high $a_3$ values.

The above scale factors may be accurately used when $(a_5/a_3)$ lies between 3.5 and 5.0, and may be conservatively used when $(a_5/a_3)$ is less than 3.5.

The seismic criteria presented herein envision that deterministic pseudo-linear seismic evaluation techniques will be used. The base acceptance capacities are defined for such an evaluation. However, it is also permissible to perform a more rigorous non-linear seismic evaluation of the underground storage tank in lieu of the simpler pseudo-linear evaluation criteria presented herein. The goal of such an evaluation is to remove unintentional conservatisms introduced by pseudo-linear simplifications. However, the intentional conservatism required in the basic criteria must be retained.

## 2 Design Basis Earthquake Ground Motion

The DBE ground motion at the site shall be defined in terms of smooth and broad frequency content horizontal and vertical response spectra defined at a specified control point location. In most cases, the control point should be the free ground surface. However, in some cases, it might be preferable to define the DBE response spectra at some other location. One such case is when a soft (less than 600 feet/second shear wave velocity), shallow (less than 50 feet) soil layer exists at the ground surface underlain by much stiffer material. In this case, the control point should be defined at the top of the stiffer material. Wherever defined, the DBE response spectra breadth and amplification should be either consistent with or conservative for the site soil profile.

The DBE response spectra may be specified using either the deterministic or probabilistic approaches described herein. Preferably, both approaches whould be used and the final DBE response spectra should be based on a combined consideration of each. In either approach, uncertainty must be appropriately considered.

### 2.1 Deterministic Approach for Specifying DBE Response Sepctra

Within the deterministic approach, one or more DBEs are specified for the site in terms of a deterministic earthquake magnitude and location. The selected DBEs should be consistent with and based upon the seismic hazard annual frequency of exceedance specified in Chapter 1 and Reference 1 (e.g., $2x10^{-4}$ annual frequency of exceedance for new high hazard facilities). For each specified DBE, approximately 84 percent non-exceedance probability (NEP) response spectra should be developed following the guidance of the U.S. Nuclear Regulatory Commission Standard Review Plan Section 2.5.2.6 (Ref. 2). The site DBE response spectra should envelope the approximately 84 percent NEP response spectra for each of the selected DBEs. The use of the 84 percent NEP response spectra provide an adequately conservative treatment of uncertainty when this deterministic approach is used.

2.2 Probabilistic Approach for Specifying DBE Response Spectra

Within the probabilistic approach, the DBE response spectra can be based on the uniform hazard spectra (UHS) associated with the seismic hazard annual frequency of exceedance specified in Section 1 and Reference 1 over the entire natural frequency range of interest (generally 0.5 to 40 Hz). Such UHS may be specified at the median (50 percent NEP) level, the 84 percent NEP level, or the mean level. Median UHS tend to be stable between different predictors and represent the midpoint ground motion estimate of the scientific community, and this situation represents an advantage to specifying the DBE response spectra at the median UHS level. However, specifying the DBE response spectra by the unfactored median UHS does not appropriately consider uncertainty. Specifying the DBE response spectra at the mean UHS level provides an appropriate consideration of uncertainty. However, mean UHS tend to be unstable between different predictors and tend to be driven by extreme upper bound models. Therefore, the following recommendation is provided:

- When stable mean UHS exist, the DBE response spectra should be set equal to the mean UHS at the appropriate seismic hazard annual frequency of exceedance. However, the use of mean UHS outside the range of 1.3 to 1.7 times the median UHS is questionable.

- Alternately, the DBE response spectra should be set at 1.5 times the median UHS at the appropriate annual frequency.

Either approach is considered to provide an adequately conservative treatment of uncertainty.

3. Analysis of Seismic Demand (Response)

It is anticipated that seismic demand will generally be estimated based upon linear response analyses. Appropriately conservative DBE response spectra (Section 2) should be used as input to such analyses. Other than for the conservatism specified in the DBE response spectra, the seismic response analyses can be median centered (no intentional conservatism), but with variation of some of the most uncertain parameters. Seismic response analyses should be conducted in accordance with the guidance contained in Reference 1 and 3 as amplified upon and modified herein.

Best estimate structural models and material damping should be used. Best estimate material damping vaues are provided in Section 4. However, approximately plus/minus one standard deviation variation in both the natural frequency of the structure model, and in soil stiffness properties should be incorporated into these analyses. In general, the structure frequency uncertainty can be accommodated by use of a 30 percent frequency uncertainty band either centered on the best estimate frequency or skewed to the low frequency side when such skewness is considered appropriate. Guidance on the appropriate variation of soil stiffness properties is given in Section 4.4.2 of Reference 1. The seismic demand, $D_S$, should be obtained from the largest computed response within these uncertainty bands. Great precision is unnecessary, and this largest response can generally be estimated from analyses that use best-estimate, upper-bound, and lower-bound properties for structure model frequency and soil stiffness.

4. Damping

Damping values recommended for dynamic analyses of moderate- and high-hazard facilities are presented in Table 2 at three different response levels. Response Level 3 corresponds to inelastic response where the elastic computed total demand (seismic plus non-seismic) exceeds the capacity limits defined herein (i.e., credit must be taken for the inelastic energy absorption factor $F_\mu$). When evaluating the component, response Level 3 damping may be used in elastic response analyses independent of the state of response actually reached. because such damping is expected to be reached prior to component failure. However, when determining the input to subcomponents mounted on the component, the component damping value to be used in elastic response analyses should be a function of the response level reached in the majority of the seismic load resisting component elements. Defining $D_T$ as the total elastic computed demand (seismic $D_S$ plus non-seismic $D_{NS}$) and $C_C$ as the component code strength capacity, then the appropriate response level damping can be estimated from the following:

| Response Level | $D_T/C_C$ |
|---|---|
| 3 | > 1.0 |
| 2 | 0.5 to 1.0 |
| 1 | < 0.5 |

The damping values presented in Table 2 are intended to be best-estimate (median centered) damping values with no intentional conservative bias for use in elastic response analyses. Other damping values may be used when such values are properly justified as best-estimate values.

Response Level 3 damping values are intended for use in elastic response analyses coupled with the permissible inelastic energy absorption factors $F_\mu$ defined later. However, when a nonlinear inelastic response analysis which explicitly incorporates the hysteretic energy dissipation is performed, no higher than response Level 2 damping values should be used to avoid the double-counting of this hysteretic energy dissipation which would result from the use of response Level 3 damping values.

5. Material Strength Properties

Material strength properties should be established at the 95 percent exceedance actual strength levels associated with the time during the service life at which such strengths are minimum. If strengths are expected to increase during the service life, then the strength of a new component should be taken as the strength at the time the component is placed in service, and the strength of an existing component should be its value at the time the evaluation is performed. If strengths are expected to degrade during the service life, then strengths to be used in the evaluation should be based upon estimated 95 percent exceedance strengths at the end of the service life.

Whenever possible, material strengths should be based on 95 percent exceedance values estimated from tests of the actual materials used at the facility. However, when such test data is unavailable, then code minimum material strengths may be used. If degradation is anticipated during the service life, then these code minimum strengths should be further reduced to account for such degradation.

6. Capacities

In general, for load combinations which include the DBE loading, capacities $C_c$ should be based upon code-specified minimum ultimate or limit-state capacity approaches coupled with material strength properties determined as defined in Section 5. For concrete, the ACI ultimate strength approach with the appropriate capacity reduction factor, $\phi$, included as specified in either ACI318 (8) or ACI349 (9) should be used. For structural steel, the AISC-LRFD (11) limit-state strength approach with the appropriate capacity reduction factor, $\phi$, included is preferred. However, the AISC-plastic design (Chapeter N, Reference 10) maximum strength approach may be used so long as the criteria of Chapter N are met. The plastic design strengths can be taken as 1.7 times the Reference 10 allowable stresses unless another factor is defined in Chapter N. For the American Society of Mechanical Engineers (ASME) pressure vessel components under ASME code jurisdiction, ASME service Level D (12) capacities should be used. In some cases, functional failure modes may require lesser limits to be defined.

In most cases, the capacity equations should be based on the most current edition of the appropriate code, particularly when the current edition is more conservative than earlier editions. However, in some cases (particularly with the ASME code, Reference 12), current code capacities may be more liberal than those specified at the time the component was designed and fabricated, because fabrication and material specification requirements have become more stringent. In these cases, current code capacities will have to be downward corrected to account for the more relaxed fabricatoin and material specifications that existed at the time of fabrication. In all cases, when material strength properties are based on code minimum material strengths, the code edition enforced at the time the component was fabricated should be used to define these code minimum material strengths.

Alternate capacity approaches may be used to establish $C_C$ when it can be demonstrated that these alternative approaches provide approximately the factor of conservatism defined by Equations (1) and (2).

## 7. Load Combinations and Acceptance Criteria

This section deals only with load combinations that include DBE loadings. In many cases, other (non-seismic) load combinations may control the design or evaluation of a component. These non-seismic load combinations should be defined by other documents.

It is assumed herein that the DBE seismic demand, $D_S$, will be computed by linear elastic analyses conducted in accordance with the response criteria defined in Sections 2 through 4. This elastic computed seismic demand $D_S$ should be modified by the appropriate inelastic energy absorption factor $F_{\mu D}$ as defined in Section 8 to obtain an inelastic factored seismic demand $D_{SI}$ by:

$$D_{SI} = \frac{D_S}{F_{\mu D}} \qquad (3)$$

The total inelastic factored demand $D_{TI}$ is then given by:

$$D_{TI} = D_{NS} + D_{SI} \qquad (4)$$

where $D_{NS}$ represents the best-estimate of all non-seismic demands expected to occur concurrent with the DBE. Equation 4 represents the DBE load-combination equation. The seismic capacity is adequate when the capacity $C_C$ determined as defined in Section 6 exceeds $D_{TI}$, i.e.,:

$$C_C \geq D_{TI} \qquad (5)$$

Equation 5 represents the seismic acceptance criterion appropriate for the DBE.

It can be demonstrated that the use of equations 3 through 5, when coupled with the appropriate $F_{\mu D}$ from Section 8, provides sufficient conservatism to achieve the seismic hazard exceedance annual frequency to seismic performance goal ratios R of 20, 10, and 5 defined in Section 1. No load factors are needed in Equation 4. The non-seismic demand $D_{NS}$ should be defined at its best-estimate level as opposed to an unlikely-to-exceed or conservative level. The conservatism embodied in defining $C_C$, $F_{\mu D}$, and $D_S$ are sufficient to achieve these R ratios without additional sources of conservatism being required.

In some cases, such as a column under combined axial compression and moment, the code capacity $C_C$ is defined in terms of interaciton equations. Furthermore, the $F_{\mu DP}$ for axial compression defined in Section 8 is less than $F_{\mu DM}$ for flexure. In this case, Equations 3 and 4 are separately entered to establish the total inelastic demands $P_{TI}$ and $M_{TI}$ for axial compression and moment, respectively, i.e.:

$$P_{TI} = P_{NS} + \frac{P_S}{F_{\mu DP}} \qquad M_{TI} = M_{NS} + \frac{M_S}{F_{\mu DM}} \qquad (6)$$

The combination of $P_{TI}$ and $M_{TI}$ is entered into the code capacity interaciton equation to determine the adequacy of the seismic design.

For ductile failure modes, non-seismic demands which are relieved by small levels of inelastic distortion (such as thermal and settlement stresses) do not have to be included in equation 4 for combination with the factored seismic demand. However, for non-ductile failure modes, these inelastic relieved non-seismic stresses must still be included. For example, if a wall capacity is controlled by flexure, these inelastic relieved non-seismic stresses don't have to be added to the seismic demand $D_{SI}$. However, if the wall capacity is controlled by shear, they do have to be added.

8. Inelastic Energy Absorption Factor

The elastic computed seismic demand $D_S$ should be factored by an inelastic energy absorption factor $F_{\mu D}$ as shown in Equation 3 to obtain a factored seismic demand $D_{SI}$. To achieve seismic hazard frequency to seismic risk frequency ratios R of 5 to 20, this inelastic energy absorption factor $F_{\mu D}$ should be defined by:

$$F_{\mu D} = 0.8\ S_F\ F_{\mu 5\%} \qquad (7)$$

where $S_F$ is the scale factor recommended in Section 1 to achieve a specified R ratio (typically $S_F$ = 1.,0, 1.2, or 1.5 for R ratios of 20, 10, or 5, respectively), and $F_{\mu 5\%}$ is the estimated inelastic energy absorption factor (inelastic seismic demand/capacity ratio) associated with a permissible level of inelastic distortions specified at about the 5 percent failure probability level.

It is always preferable to perform nonlinear analysis on the structure or component being evaluated in order to estimate $F_{\mu 5\%}$ for use in Equation 7 to define $F_{\mu D}$. However, such analyses are often costly and controversial. Therefore, a set of standard $F_{\mu D}$ values is provided in Table 3 for common elements associated with underground hazardous waste storage tanks, and discussed in the remainder of this section. Additional $F_\mu$ values are provided in Table 4-7 of Reference 1. The $F_\mu$ presented in Reference 1 for the high hazard category may be used for $F_{\mu D}$ when the applicable scale factor $S_F$ is 1.0, and the $F_\mu$ presented in Reference 1 for the moderate hazard category correspond to $F_{\mu D}$ for $S_F$ of 1.2 to 1.25. The $F_{\mu D}$ values presented in Table 3 or from Reference 1 may be used in lieu of performing nonlinear analyses, so long as the following cautions are observed.

The use of $F_{\mu D}$ values listed in Table 3 corresponding to $F_{\mu 5\%}$ = 1.5 and greater for concrete walls is conditional on extensive wall cracking, but stable wall behavior constituting acceptable wall performance. If only minor wall cracking is acceptable, the $F_{\mu D}$ should be based on $F_{\mu 5\%}$ = 1.0.

The $F_{\mu D}$ values listed in Table 3 for ductile failure modes ($F_{\mu 5\%}$ greater than 1.0) assume that steel reinforcing bars, metal tank shells, and anchorage will remain ductile during the component's entire service life. It is assumed that the metal will retain at least a 10 percent uniaxial elongation strain capability. If this metal can become embrittled at some time short of the end of the service life, $F_{\mu D}$ values should be based on $F_{\mu 5\%}$ = 1.0.

For low-ductility failure modes such as axial compression or shear in concrete walls or columns and wall-to-diaphragm, wall-to-column, or column-to-base connections, the $F_{\mu D}$ values listed in Table 3 correspond to $F_{\mu 5\%} = 1.0$ In most cases, such stringent limits can be relaxed somewhat, as described below. Unless the ductile flexural failure mode has seismic capacity significantly in excess of its required capacity, inelastic distortions in this ductile failure mode will likely limit the factored seismic demand $D_{SI}$ in the low-ductility failure modes to levels less than those given by Equation 3 with the $F_{\mu D}$ values given by Table 3. Because greater conservatism exists in code capacities $C_C$ for low-ductility failure modes than exists in $C_C$ for ductile failure modes, the failure will be controlled by the ductile failure mode so long as the low-ductility failure mode code capacity is at least equal to the ductile failure mode capacity. Thus, for low-ductility failure modes, the factored seismic demand $D_{SI}$ can be limited to the lesser of:

1. $D_{SI}$ given by Equation 3 using $F_{\mu D}$ from Table 3, or

2. $D_{SI} = C_C - D_{NS}$ computed for the ductile failure mode, where $C_C$ is the ductile failure mode code capacity.

Therefore, for example, connections do not have to be designed to have code capacities $C_C$ greater than the code capacities $C_C$ of the members being connected, or the total factored demand $D_{TI}$ given by Equations 3 and 4 and Table 3, whichever is less. Similarly, the code shear capacity of a wall does not have to exceed the total shear load which can be supported by the wall at the code flexural capacity of the wall. Finally, horizontal seismic-induced axial force in a moment frame column can be limited to the axial force which can be transmitted to the column when a full plastic hinge mechanism develops in the frame where the plastic hinge capacities are defined by the code ultimate flexural capacities.

When the dominant response frequency (generally the fundamental frequency) of the component lies to the stiff (high frequency) side of the frequency at which the input spectral acceleration is maximum, the cautions contained in Section 4.2.3 and illustrated in Figure 4-4 of Reference 1 remain in effect. In this case, the factored seismic demand $D_{SI}$ may be computed from either of the following two approaches:

1. The maximum input spectral acceleration shall be used to compute the elastic response of the dominant response mode in lieu of the lesser input spectral acceleration corresponding to the modal frequency of this mode. In this case, the $F_{\mu D}$ values listed in Table 3 or Reference 1 may be used.

2. The lesser input spectral acceleration corresponding to the modal frequency of the dominant response mode may be used together with $F_{\mu D}$ values based on $F_{\mu 5\%}$ = 1.0, or $F_{\mu 5\%}$ computed from nonlenear analyses.

Reference


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Table 1 Basic Seismic Criteria

| Usage Category | Design Basis Earthquake (DBE) Annual Frequency of Exceedance | Facility Type | Performance Goal Annual Freq. of Exceedance | Reduction Factor | Scale Factor $S_f$ |
|---|---|---|---|---|---|
| High Hazard | $2x10^{-4}$ | New | $1x10^{-5}$ | 20 | 1.0 |
| | | Existing (When Justified) | $2x10^{-5}$ | 10 | 1.2 |
| Moderate Hazard | $1x10^{-3}$ | New | $1x10^{-4}$ | 10 | 1.2 |
| | | Existing (When Justified) | $2x10^{-4}$ | 5 | 1.50 |

Table 2

Recommended Damping Values
(Based on Ref. 1, 3, 4, 5, 6, and 7)

| Type of Component | Damping (% of Critical | | |
|---|---|---|---|
| | Response Level 1 | Response Level 2 | Response Level 3 |
| Welded and friction-bolted steel structures | 2 | 4 | 7 |
| Bearing-bolted steel structures | 4 | 7 | 10 |
| Prestressed concrete structures (without complete loss of prestress) | 2 | 5 | 7 |
| Reinforced concrete structures | 4 | 7 | 10 |
| Masonry shear walls | 4 | 7 | 12 |
| Wood structures | 7 | 10 | 15 |
| Piping | 3 | 5 | 5 |
| Massive, low-stressed components (pumps, motors, etc.) | 2 | 3 | --(1) |
| Light, welded instrument racks | 2 | 3 | --(1) |
| Electrical cabinets | 3 | 4 | 5 |
| Liquid containing metal tanks: | | | |
| Impulsive mode | 2 | 3 | 3 to 5(2) |
| Sloshing mode | 0.5 | 0.5 | 0.5 |

(1) Should not be stressed to response level 3.

(2) For tanks where total mass is dominated by fluid mass and one cannot identify sources of significant hystertic energy dissipation due to nonlinear behavior in supports, anchorage, etc., or energy feedback into the foundation media, 3% damping is appropriate. If significant sources of hysteretic energy dissipation or energy feedback can be identified and are not explicitely included in the analysis, then 5% damping can be used to represent these effects.

Table 3

Inelastic Energy Absorption Factors
(Inelastic Demand/Capacity Ratios)
$F_{\mu D}$

| Structural System | $F_{\mu 5\%}$ | Scale Factor $S_F$ 1.5 | 1.2-1.25 | 1.0 |
|---|---|---|---|---|
| Concrete Vault | | | | |
| Walls: | | | | |
| In-plane: | | | | |
| Flexure | 1.75 | 2.1 | 1.7 | 1.4 |
| Shear | 1.5 | 1.8 | 1.5 | 1.2 |
| Out-of-plane: | | | | |
| Flexure | 1.75 | 2.1 | 1.7 | 1.4 |
| Shear | 1.0 | 1.2 | 1.0 | 0.8 |
| Columns: | | | | |
| Axial Compression | 1.0 | 1.2 | 1.0 | 0.8 |
| Flexure | 1.5 | 1.8 | 1.5 | 1.2 |
| Shear | 1.0 | 1.2 | 1.0 | 0.8 |
| Connections | 1.0 | 1.2 | 1.0 | 0.8 |
| Metal Liquid-Storage Tanks | | | | |
| Moment and Shear Capacity | 1.25 | 1.5 | 1.2 | 1.0 |
| Hoop Capacity | 1.5 | 1.8 | 1.5 | 1.2 |

**Figure 1. Typical Probabilistic Seismic Hazard Curve**